

Modelling hollowcore concrete slabs subjected to fire

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ABSTRACT: Precast, prestressed hollow core concrete slabs have been widely used in recent years. In a previous paper at ACMSM19 the authors presented a simple computational method that could be used in design for modelling the structural behaviour of hollowcore concrete slabs in fires. The model proposed at that time consisted of a grillage system using beam elements to include the thermal expansion in both directions and to simulate the vertical cracking in the flanges, with the topping concrete modelled using shell elements. The simulation showed good agreement with the experimental results. This present paper shows that it is possible to avoid using shell elements to model the topping concrete and instead model the topping concrete as part of the hollowcore slabs. This beam grillage method of modelling both hollowcore slabs and topping concrete gives similar results to using beam and shell elements, but requires less computational time.

1 INTRODUCTION

Precast, prestressed hollowcore concrete floors are very popular in multi-storey buildings because of their excellent structural performance in ambient conditions, high quality control and low on-site labour costs. Hollowcore concrete floors are designed as one-way slab systems, with the units sitting side-by-side, spanning between supporting walls or beams. Most hollowcore concrete (HC) floors have in-situ reinforced concrete topping. Structural behaviour of hollowcore concrete floors is dominated by action parallel to the units and their prestressing strands. Two-way action can sometimes occur in such slab systems, resulting from transverse structural behaviour of the topping concrete, depending on the vertical supports parallel to the hollowcore units (Fellinger 2004; Fib 1998; BEF 2005; Chang et al 2008a).

The structural behaviour of a HC floor system under fire is complicated. There are many existing studies investigating this behaviour using different approaches, and precise computer models have been developed to improve the understanding of such systems (Fellinger 2004). However, very detailed finite element analyses of the structural fire behaviour of HC slabs are too time-consuming to apply in the everyday design process. On the other hand, simplistic approaches using simple code rules are insufficient to capture the effects of the support conditions. Consequently, a simple yet sufficiently accurate computational method for designers to

model the structural behaviour of HC slabs under fire needs to be developed.

This paper investigates the possibility of modeling the HC slabs and the topping solely by longitudinal and transverse beam elements.

2 BACKGROUND

The behaviour of HC slabs under fire is more complicated than that of solid slabs. The voids cause discontinuity in heat transfer, yet the thermal gradient needs to be addressed correctly to accurately model the temperature induced mechanical strains occurring in the webs. The support conditions also have significant influence on the structural behaviour and should be considered in design. The presence of prestressing stress can considerably influence the predicted overall structural performance (Chang 2007), as the HC units have no reinforcing and the resistance to tensile stresses come from the prestressing tendons.

In a previous paper, Chang et al (2006) showed that the performance of hollowcore floor systems in fire can be successfully predicted by using a grillage of 3D beam elements to simulate the hollowcore units and a layer of shell elements to represent the topping concrete slab which covers the hollowcore units and connects the hollowcore units to each other and to the surrounding structural members (Fig. 1).

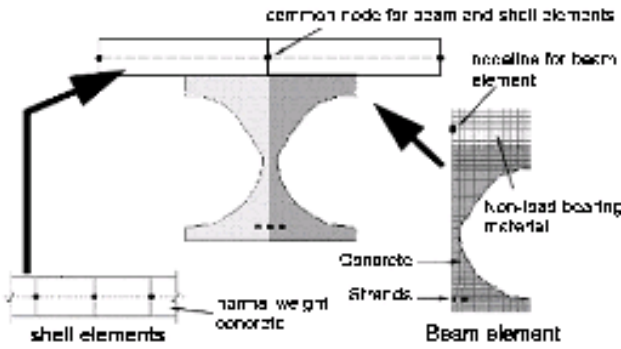


Figure 1. Discretisation of the cross section of hollowcore unit in the original method.

Most importantly, the results from this modelling method showed good agreement with experimental results available in literature [Chang et al 2008a].

The model developed in the previous study worked well for small subassemblies. However, although the sections representing the topping in the beam elements needed in the thermal analysis (Fig. 1) do not contribute to the performance of the slab, they are modelled as non-load bearing material and still consume a lot of computer resources in the structural analysis. As a result, the model becomes too complicated when analysing subassemblies containing more than 4 parallel hollowcore units.

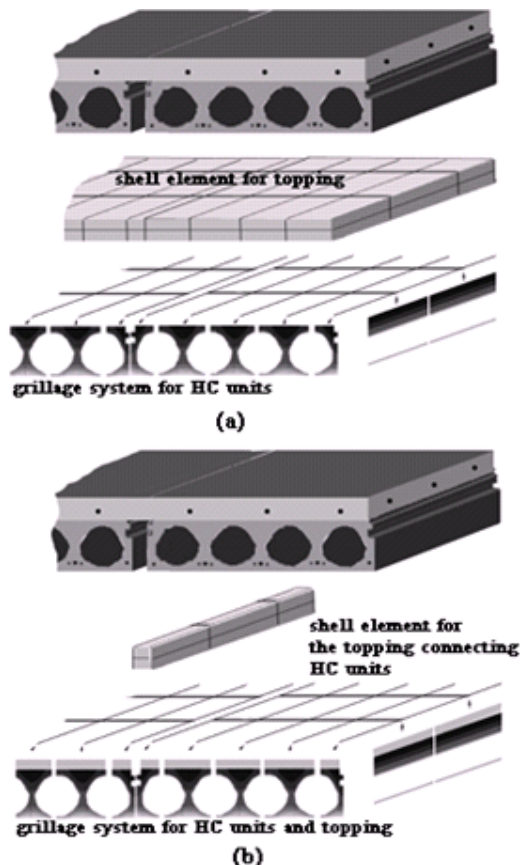


Figure 2. Schematic drawing for the (a) original (b) second method to model hollowcore floor systems.

Therefore, a different model was needed in order to study the effect of aspect ratio on the fire performance of hollowcore concrete floor systems (Chang et al 2008b). The schematic drawings of the two modelling methods are shown in Figure 2. It was found during the development of the original model that, when modelling the floor slab with only one hollowcore unit, simulating the topping slab as part of the beam elements or separately by the shell elements gave the same result (Chang 2007). Hence, instead of giving the section representing the topping in the beam elements zero strength and using shell elements to simulate the topping, the topping can be modelled as part of the beam elements and nearly all the shell elements can be removed from the model.

This method worked well and was used for the studies reported in Chang et al (2008b), using 300 mm deep hollowcore units. Present studies involve 200 mm deep units which contain six hollow cores and need to be represented by seven beam elements interconnected by transverse elements to provide continuity within the hollowcore units.

3 MODELLING OF HOLLOWCORE SLABS IN SAFIR

The details and dimensions of the hollowcore floor slab investigated are shown in Figure 3. A typical 200 mm deep prestressed hollowcore unit is shown in Figure 4 while the properties of such units are shown in Table 1.

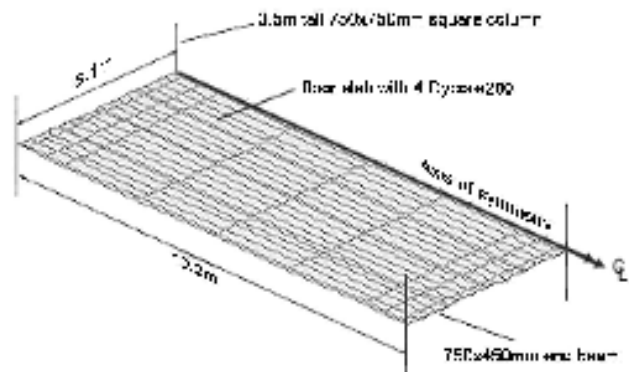


Figure 3. Dimensions and layout of the studied subassembly.

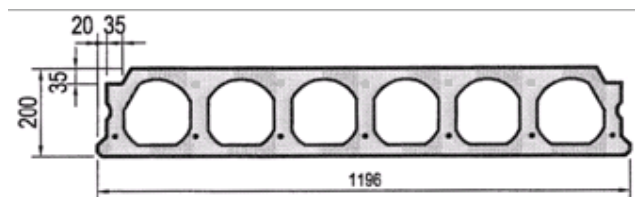


Figure 4. Cross-section of a 200 mm hollowcore unit.

Table 1. Properties of the hollowcore floor system.

200 hollowcore	
Cross-sectional area	0.121 m ²
Self weight	2.61 kPa
Compressive strength	45 MPa
Prestressing strands	
Type	Stress relieved 7-wire strand
Strength	1.87 GPa
Prestressing level	70%
Cross-sectional area/strand	112 mm ²
Reinforced concrete topping slab	
Concrete compressive strength	30 MPa
Reinforcement strength	450 MPa

The analytical simulations have been carried out using SAFIR (Franssen 2007), a non-linear finite element analysis program which is able to carry out both structural and thermal analysis, with thermal and mechanical properties from Eurocodes 2 (EC2 2002) and 3 (EC3 2002) integrated into the program. The standard ISO fire (1999) was used in all the fire simulations reported herein. Additionally, SAFIR was used to investigate the ultimate strength of the HC floors at ambient temperature.

The beam grillages to model the hollowcore units and the topping concrete are illustrated in Figure 5. Each HC unit is considered to be comprised of five

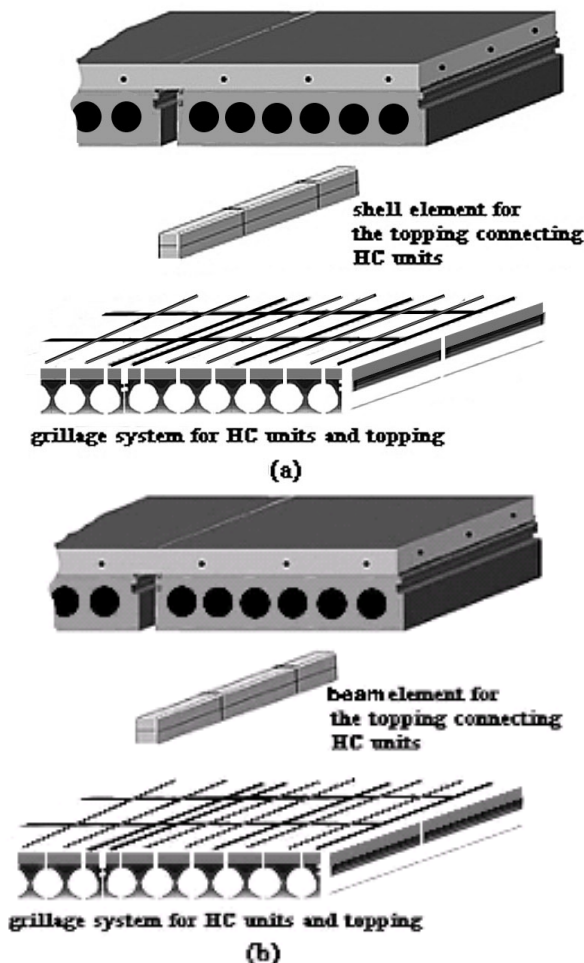


Figure 5. Schematic drawing for the 200 mm HC floors using (a) shell elements, and (b) beam elements to connect individual HC units.

'I' shaped sections that are symmetric about the web centerline and two non-symmetric edge members. The transverse members within each unit comprise only the top and bottom flanges, plus topping concrete.

The HC units are supported at each end on beams as illustrated in Figure 6(a). The reaction between the HC unit and the beam is offset from the centre of rotation of the beam while the nodeline for all the beams was taken as being at the mid-depth of the topping concrete. Figure 6(b)-(d) illustrate the end connection modeling when using shell elements for the topping concrete in the original model (Chang et al 2006), using shell elements only for the inter-HC unit connection (Chang et al 2008a), and beam elements only.

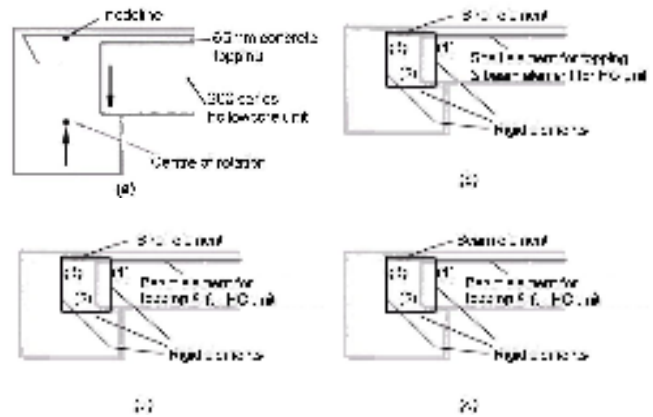


Figure 6. Schematic drawing of the HC end support modelling.

In the beam grillages, the longitudinal beams run in the direction of the span and represent the webs and flanges of the hollowcore units. In SAFIR, the prestressing effect in the longitudinal beam is considered by including it when calculating the stress equilibrium in the first time step of the structural analysis. The transverse beams in the grillage model both the top and bottom flanges and run in the direction across the hollowcore unit. These transverse beams are able to capture the effect of thermal expansion in the transverse direction of each hollowcore unit. In the thermal analysis the topping is included in both longitudinal and transverse beams to calculate the thermal gradient correctly, but as the topping is simulated using shell elements in the structural analysis, the section representing the topping in the beam elements in the thermal analysis is taken as an arbitrary material without strength or stiffness. In the shell elements, the reinforcing bars in the topping slab are simulated as layers of smeared steel section across the shell element with each layer exhibiting a uniaxial behaviour.

This modelling scheme does not consider shear and anchorage failures. As perfect bond between the

concrete and the reinforcing steel is assumed for both beam and shell elements, as well as between the topping slab (shell elements) and the hollowcore units, bond failures are also not accounted for. It also does not consider spalling or the vertical tensile stresses in the web of hollowcore units. Nevertheless, the model considers the prestressing effect, the thermal strains as well as the mechanical stresses induced by incompatible thermal strains in both lateral and longitudinal directions, and the continuity between the hollowcore units which subsequently allows the model to take account of the effects of the end and side supports.

4 COMPARISON OF RESULTS

Two simulation methods were compared: the first method has the topping on the HC units included as part of the beam element and that part of the topping connecting the HC units together modelled by shell elements, while the second method replaces the shell elements by further beam elements. In both cases, the modelled floor is as shown in Figure 3 and has been analysed both with and without side beam support.

Figure 7 shows the deflected shape of an HC floor without side beam support; the only difference between the two models, and whether or not a fire or an ultimate load analysis has been carried out, is the magnitude of the resulting deflections. The deflected shape of a side beam supported floor is illustrated in Figure 8.

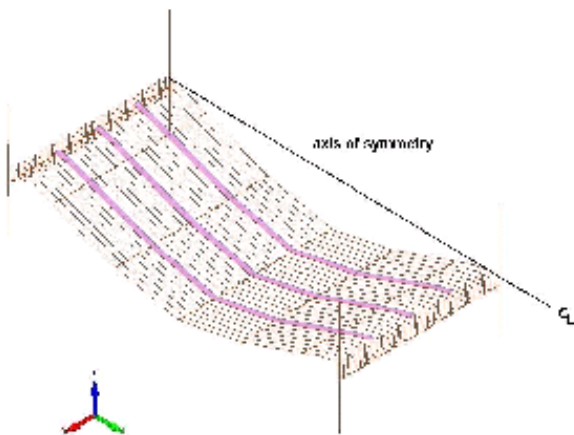


Figure 7. Typical deflected shape of an HC floor without side beam support.

Figure 9 shows a comparison of the results for the cases of increasing load under ambient temperature where a load factor of 1 represents the design fire load of 5.24 kPa being (DL+0.4LL). For the case without edge beams, the two simulations are in

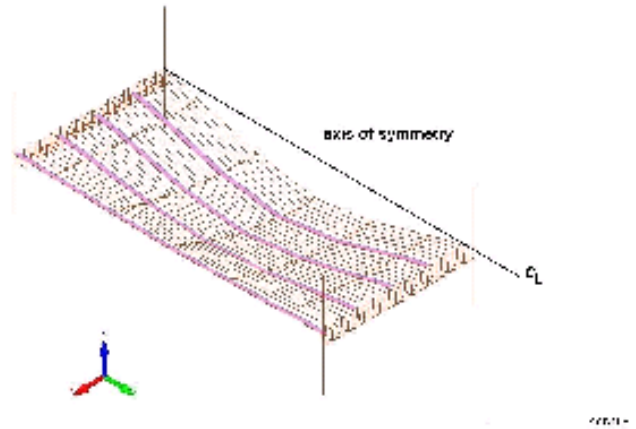


Figure 8. Typical deflected shape of an HC floor with side beam support.

reasonably close agreement up to failure at a load factor of about 4.5. It can be seen that the use of beams only leads to a slightly more flexible model. When side beam restraint is present, the two modeling methods give very similar results up to a load factor of about 5.5 before diverging slightly, with the presence of the shell elements appearing to provide a greater strength prediction and a slightly more flexible model.

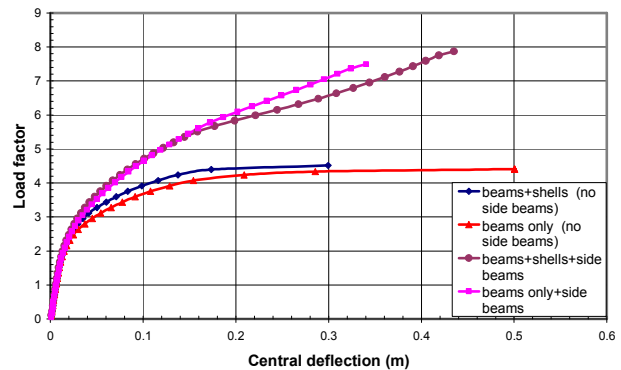


Figure 9. Comparison of the ultimate load-deflection behaviour predicted by the two simulation methods.

Figure 10 shows a comparison of the deflections at mid-span predicted by the two methods. While they show the same trend for the case with no side beams, the method using only beam elements to model the floor appears to be slightly more flexible than the method wherein shell elements are used to make the connection between the individual HC units. In the case where side beams are present, the two analyses, with and without shell elements, are initially similar but soon diverge with the method using only beam elements being stiffer than the method using shell elements to represent the topping between HC units.

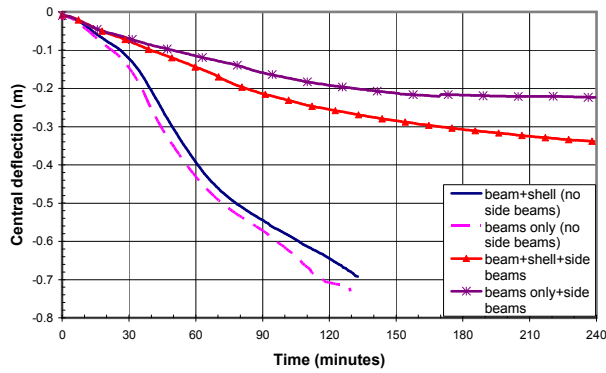


Figure 10. Comparison of the mid-span deflection from the two simulation methods

5 DISCUSSION

The numerical analyses shown in Figure 10 indicate that the HC floors can resist over 120 minutes of ISO fire heating before failing, and this supports the hollowcore manufacturer's statement that the HC units have a two hour fire resistance rating. This does not imply that the floors would necessarily survive ANY particular 2 hour fire as a fire that burns out, then cools slowly, will affect the floor in a different manner to a standard ISO fire.

From Figure 7 it can be seen that when no side beam restraints are present, the floor behaviour is essentially that of the individual HC units. The load ratio during the fire is the ratio of the design fire load to the ultimate load of the member. For the HC floors analysed herein, the load ratio is the inverse of the load factor and is about 22% of the ultimate strength determined by the analyses. When side beam are present to restrain deflections at the sides of the floor, the load factor is in the range of 7.5-8 and the load carried by the floor during the fire is about 13% of the ultimate. Looked at another way, the floor behaves as a one-way spanning floor system when no side beams are present, and the strength in a fire is greatly increased when side beams are present to provide two way action through the topping concrete.

6 CONCLUSIONS

From the above analyses, it appears that the use of only beam elements to model both the HC units and the topping concrete provides similar results to using beam elements for the HC units and using shell elements to represent the topping concrete connecting individual HC units.

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